Chapter 2
Causes of Structural Deterioration

2-1. Corrosion

a. Effects of corrosion. Corrosion can seriously weaken a structure or impair its operation, so the effect of corrosion on the strength, stability, and serviceability of hydraulic steel structures must be evaluated. The major degrading effects of corrosion on structural members are a loss of cross section, buildup of corrosion products at connection details, and a notching effect that creates stress concentrations.

(1) A loss of cross section in a member causes a reduction in strength and stiffness that leads to increased stress levels and deformation without any change in the imposed loading. Flexure, shear, and buckling strength may all be affected. Depending on the location of corrosion, the percentage reduction in strength considering these different modes of failure is not generally not the same.

(2) A buildup of corrosion products can be particularly damaging at connection details. For example, corrosion buildup in a tainter gate trunnion or lift gate roller guides can lead to extremely high hoist loads. At connections between adjacent plates or angles, a buildup of rust can cause prying action. This is referred to as corrosion packout and results from expansion during the corrosion process.

(3) Localized pitting corrosion can form notches that may serve as fracture initiation sites. Notching significantly reduces the member fatigue life.

b. Common types of corrosion. Corrosion is degradation of a material due to reaction with its environment. All corrosion processes include electrochemical reactions. Galvanic corrosion, pitting corrosion, crevice corrosion, and general corrosion are purely electrochemical. Erosion corrosion and stress corrosion, however, result from the combined action of chemical plus mechanical factors. In general, hydraulic steel structures are susceptible to three types of corrosion: general atmospheric corrosion, localized corrosion, and mechanically assisted corrosion (Slater 1987). For any case, the type of corrosion and cause should be identified to assure that a meaningful evaluation is performed.

(1) General atmospheric corrosion is defined as corrosive attack that results in uniform thinning spread over a wide area. It is expected to occur in the ambient environment of hydraulic steel structures but is not likely to cause significant structural degradation.

(2) Localized corrosion is the type of corrosion most likely to affect hydraulic steel structures. Five types of localized corrosion are possible:

(a) Crevice corrosion occurs in narrow openings between two contact surfaces, such as between adjoining plates or angles in a connection. It can also occur between a steel component and a nonmetal one (under the seals, a paint layer, debris, sand or silt, or organisms caught on the gate members). It can lead to blistering and failure of the paint system, which further promotes corrosion.

(b) Pitting corrosion occurs on bare metal surfaces as well as under paint films. It is characterized by small cavities penetrating into the surface over a very localized area (at a point). If pitting occurs under paint, it can result in the formation of a blister and failure of the paint system.

(c) Galvanic corrosion can occur in gate structures where steels with different electrochemical potential (dissimilar metals) are in contact. The corrosion typically causes blistering or discoloration of the paint and failure of the paint system adjacent to the contact area of the two steels and decreases as the distance from the metal junction increases.

(d) Stray current corrosion may occur when sources of direct current (i.e., welding generators) are attached to the gate structures, or unintended fields from cathodic protection systems are generated.

(e) Filiform corrosion occurs under thin paint films and has the appearance of fine filaments emanating from one or more sources in random directions.

(3) Three types of mechanically assisted corrosion are also possible in hydraulic steel structures.

(a) Erosion corrosion is caused by removal of surface material by action of numerous individual impacts of solid or liquid particles and usually has a direction associated with the metal removal. The precursor of erosion corrosion is directional removal of the paint film by the impacting particles.

(b) Cavitation corrosion is caused by cavitation associated with turbulent flow. It can remove surface films such as oxides or paint and expose bare metal, producing rounded microcraters.

(c) Fretting corrosion is a combination of wear and corrosion in which material is removed between contacting surfaces when very small amplitude motions occur between the surfaces. Red rust is formed and appears to come from between the contacting surfaces.

c. Factors influencing corrosion. The type and amount of corrosion that may occur on a hydraulic steel structure are dependent on many factors that include design details, material properties, maintenance and operation, environment, and coating system. In general, the primary factors are the local environment and the protective coating system.

(1) The pH and ion concentration of the river water and rain are significant environmental factors. Corrosion usually occurs at low pH (highly acidic conditions) or at high pH (highly alkaline conditions). At intermediate pH, a protective oxide or hydroxide often forms. Deposits of film-forming materials such as oil and grease and sand and silt can also contribute to corrosion by creating crevices and ion concentration cells.

(2) Corrosion of steel increases significantly when the relative humidity is greater than 40 percent. Corrosion is also aggravated by alternately wet and dry cycles with longer periods of wetness tending to increase the effect. Organisms in contact with steel also promote corrosion.

(3) Paint and other protective coatings are the primary preventive measures against corrosion on hydraulic steel structures. The effectiveness of a protective coating system is highly dependent on proper pretreatment of the steel surface and coating application. Sharp corners, edges, crevices, weld terminations, rivets, and bolts are often more susceptible to corrosion since they are more difficult to coat adequately. Any variation in the paint system can cause local coating failure, which may result in corrosion under the paint.

(4) The paint system and cathodic protection systems should be inspected to assure that protection is being provided against corrosion. If corrosion has occurred, ultrasonic equipment and gap gauges are available to measure loss of material.

2-2. Fracture

a. Basic behavior.

(1) Brittle fracture is a catastrophic failure that occurs suddenly without prior plastic deformation and can occur at nominal stress levels below the yield stress. Fracture of structural members occurs when a relatively high stress level is applied to a material with relatively low fracture toughness.

(2) Fracture usually initiates at a discontinuity that serves as a local stress raiser. Structural connections that are welded, bolted, or riveted are sources of discontinuities and stress concentrations because members are discontinuous and abrupt changes in geometry occur where different members intersect. Welded connections include additional physical discontinuities, metallurgical structure variations, and residual stresses that further contribute to possible fracture. The fracture or cracking vulnerability of a structural component is governed by the material fracture toughness, the stress magnitude, the component geometry, and the size, shape, and orientation of any existing crack or discontinuity (see *b* and *c* below).

b. Fracture mechanics concepts.

(1) Fracture mechanics includes linear-elastic fracture mechanics (LEFM) and elastic-plastic fracture mechanics (EPFM). In LEFM analysis, it is assumed that the material in the vicinity of a crack tip is linear-elastic. EPFM methods, which include the crack tip opening displacement (CTOD) and J-integral methods, take into account plastic material behavior. Some fundamental concepts of LEFM are presented here. Additional information is provided in Chapter 6, and examples applying this methodology to hydraulic steel structures are located in Chapter 7.

(2) When tensile stresses are applied to a body that contains a discontinuity such as a sharp crack, the crack tends to open and high stress is concentrated at the crack tip. For cases where plastic deformation is constrained to a small zone at the crack tip (plane-strain condition), the fracture instability can be predicted using LEFM concepts. The fundamental principle of LEFM is that the stress field ahead of a sharp crack in a structural member can be characterized in terms of a single parameter, the stress intensity factor K_I. K_I is a function of the crack geometry and nominal stress level in the member, and K_I has the general form

$$K_I = C\sigma\sqrt{a} \qquad (2\text{-}1)$$

where

C = nondimensional coefficient that is a function of the component and crack geometry

σ = member nominal stress

a = crack length

K_I is in units of Mpa-$\sqrt{\text{m}}$ (ksi-$\sqrt{\text{in.}}$) and, for a given crack size and geometry, is directly related to the nominal stress.

(3) Another basic principal of LEFM is that fracture (unstable crack propagation) will occur when K_I exceeds the critical stress intensity factor K_{Ic} (or K_c depending on the state of stress at the crack tip). K_{Ic} represents the fracture toughness (ability of the material to withstand a given stress-field intensity at the tip of a crack and to resist tensile crack extension) of a component when the state of stress at the crack tip is plane strain and the extent of yielding at the crack tip is limited. This is generally the case for relatively thick

sections where a triaxial state of stress exists (due to the constraint in the through thickness direction) at the crack tip. Plane strain behavior occurs when

$$\beta_{Ic} = \frac{1}{t}\left(\frac{K_{Ic}}{\sigma_y}\right)^2 \leq 0.4 \qquad (2\text{-}2)$$

where

β_{Ic} = Irwin's plane strain factor

t = thickness of the component

K_{Ic} = critical plane strain stress intensity factor

σ_y = yield stress

(4) K_{Ic} is a material property (for a given temperature and loading rate) that is defined by American Society for Testing and Materials (ASTM) E399 and is applicable only when plane strain conditions exist. When this requirement for plane strain conditions is not met, the fracture toughness of a component may be defined by the critical stress intensity factor K_c. K_c is the fracture toughness under other than plane strain conditions and is a function of the thickness of the component in addition to temperature and loading rate. K_c is always greater than K_{Ic}.

(5) For many structural applications where low- to medium-strength steels are used, the material thickness is not sufficient to maintain small crack-tip plastic deformation under slow loading conditions at normal service temperatures. Consequently, the LEFM approach is invalidated by the formation of large plastic zones and elastic-plastic behavior in the region near the crack tip. When the extent of yielding at the crack tip becomes large, EPFM methods are required. One widely used EPFM method is the CTOD method of fracture analysis (British Standards Institution 1980). The CTOD method is more applicable when there is significant plastification, since it is a direct measurement of opening displacement and is not based on calculated elastic stress fields. The LEFM and CTOD methods are discussed further in Chapter 6.

c. Factors influencing fracture. Many factors can contribute to fracture and weld-related cracking in hydraulic steel structures. These include material properties (fracture toughness), welding influences, and component thickness.

(1) Material properties. Material fracture toughness of steel is generally a function of chemical composition, thermomechanical history, and microstructure. Chemical composition affects the toughness of a steel, since the addition of solute (e.g., alloying and/or tramp elements) to a metal may inhibit plastic flow, which strengthens the material, but reduces its fracture toughness. Thermomechanical treatment can affect toughness by altering the phase composition of the material. The microstructure, particularly the grain size, also affects the fracture toughness. For a given steel, fracture toughness will generally tend to decrease with increasing grain size much the same as yield strength does. Fracture toughness will also vary significantly with temperature and loading rate (see Chapter 6). Structural steels exhibit a transition from a brittle behavior to a more ductile behavior at a certain temperature that is material dependent. Steel is also strain-rate sensitive, and fracture toughness decreases with increasing loading rate.

(2) Welding influences.

(a) Weld-related cracking is a result of welding discontinuities, residual stresses, and decreased strength and toughness in the weld metal and heat-affected zone (HAZ). Design and fabrication methods also affect weld integrity. Stress concentrations from notches, residual stresses, and changes in microstructure resulting in reduced toughness can also be caused by flame cutting.

(b) Common weld discontinuities such as porosity, slag inclusion, and incomplete fusion (see Chapter 4) serve as local stress concentrations and crack nucleation sites. Discontinuities in regions near the weld are of special concern, since high tensile residual stresses develop from the welding process.

(c) During welding, nonlinear thermal expansion and contraction of weld and base metal produce significant residual stresses. Near the weld, high tensile residual stresses may cause cracking, lamellar tearing in thick joints, and premature fracture of the welded connection. These stresses can also indirectly cause cracking by contributing to a triaxial stress state that tends toward brittle behavior. For example, at weld intersections (such as the corner of a girder flange, web, and transverse stiffener) a high triaxial state of residual tensile stress exists that is conducive to crack initiation and brittle fracture. (This detail can be improved using a coped stiffener or by not welding the stiffener to the flange.) The heat applied during the welding process also alters the microstructure in the vicinity of the weld or HAZ, which results in reduced toughness and strength in this area.

(d) Welded details that have poor accessibility during fabrication are prone to cracking due to the increased difficulty in producing a sound weld. Tack welds used for positioning and alignment of components during the fabrication can be a source of problems, since they are not usually inspected and may include significant weld discontinuities and residual stresses. This may be especially true of welds on riveted structures, since the structural steels typically used in older structures are not characterized as steels for welding. A discussion of structural steels used in older spillway gates is provided in Chapter 7. Backup bars may also be a source of discontinuity if they are not welded continuously.

(3) Thick plates. Thick plate material tends to be more susceptible to cracking, since during manufacturing the interior of a thick plate cools more slowly after rolling than that of a thin plate. Slow cooling of steel results in a microstructure with large grain size, and consequently, reduced toughness. The additional through-thickness constraint inherent in thick material also contributes to the susceptibility of cracking by promoting plane strain behavior. Weldments involving thick plates are particularly more susceptible to cracking than those of thin plates. In addition to the reduced toughness and additional through-thickness constraint inherent in thick plates, welding further increases the likelihood of cracking. Residual stresses due to welding are generally higher for weldments of increasing plate thickness simply because the increased thickness provides more constraint to weld shrinkage. Additionally, thick plate weldments require more weld passes so the number of thermal cycles (heating and cooling) and the probability of forming discontinuities increase. Another consideration for thick plate weldments is that a weld of a particular size will cool faster on a thick plate than a thin plate. Rapid cooling of the weld material and HAZ promotes the formation of martensite, which is a brittle phase of steel. Preheat and postheat requirements have been adopted (American National Standards Institute/American Welding Society (ANSI/AWS) D1.1) to minimize this effect.

2-3. Fatigue

Fatigue is the process of cumulative damage caused by repeated cyclic loading. Fatigue damage generally occurs at stress-concentrated regions where the localized stress exceeds the yield stress of the material. After a certain number of load cycles, the accumulated damage causes the initiation and propagation of a crack. Although the number of load cycles experienced by hydraulic steel structures does not, in general, compare to that of bridges, fatigue is a real concern for lock gates at busy locks and spillway gates with vibration problems.

a. Basic behavior.

(1) Like brittle fracture, fatigue cracking occurs or initiates at a discontinuity that serves as a stress raiser. Consequently, there are some parallels in the analysis of fatigue and fracture. Fatigue crack propagation is related to the stress intensity factor range ΔK, which serves as the driving force for fatigue (analogous to K_I considering fracture). More detailed information on fatigue crack propagation is given in Chapter 6. Here, the concept of fatigue life is introduced and will later be used to identify critical connections in Chapter 3.

(2) The fatigue life of a connection or detail is commonly defined as the number of load cycles that causes cracking of a component. The most important factors governing the fatigue life of structures are the severity of the stress concentration and the stress range of the cyclic loading. The fatigue life of a structure (member or connection) is often represented by an S_r-N curve, which defines the relationship between the constant-amplitude stress range S_r (σ_{max} - σ_{min}) and fatigue life N (number of cycles), for a given detail or category of details. The effect of the stress concentration for various details is reflected in the differences between the S_r-N curves. The S_r-N curves are based on constant-amplitude cyclic loading and are typically characterized by a linear relationship between $\log_{10} S_r$ and $\log_{10} N$. There is also a lower bound value of S_r, known as the fatigue limit, below which infinite life is assumed.

b. Fatigue strength of welded structures.

(1) Common welded details have been assigned fatigue categories (A, B, B', C, D, E, and E') and corresponding S_r-N curves. These curves have been derived from large amounts of experimental data and have been verified with analytical studies. S_r-N curves for welded details adopted by American Association of State Highway and Transportation Officials (AASHTO) for redundant structural members (AASHTO 1996) are shown in Figure 2-1. The dashed lines in Figure 2-1 represent the fatigue limit of the respective categories. Fatigue category A represents plain rolled base material and has the longest life for a given stress range and the highest fatigue limit. Categories B through E' represent increasing severity of stress concentration and associated diminishing fatigue life for a given stress range. Descriptions and illustrations of various welded details and their fatigue categories are given in Table 2-1 and Figure 2-1 (AASHTO 1996).

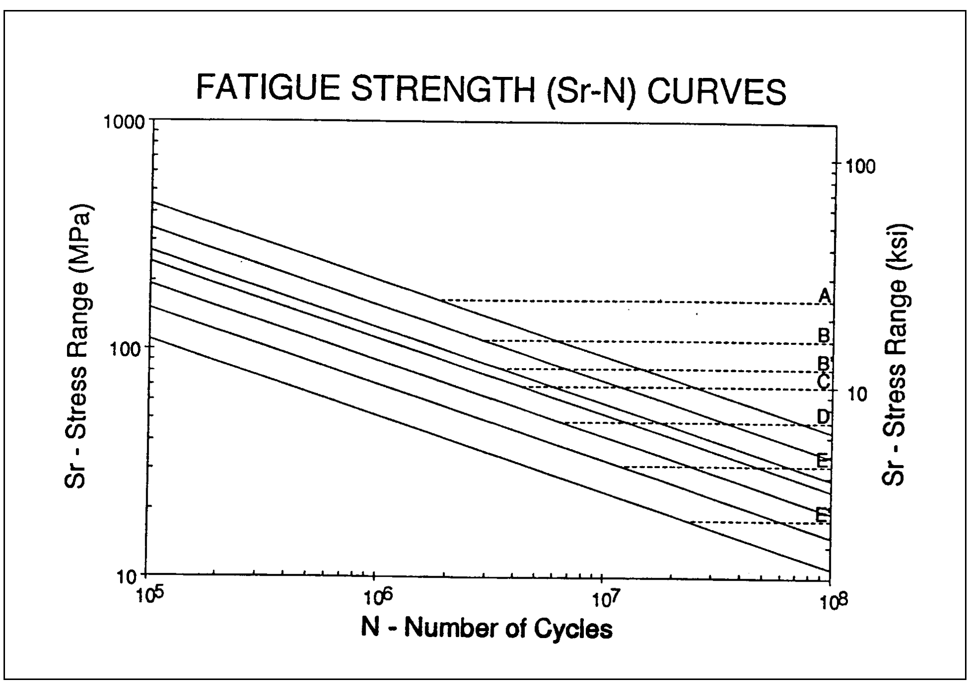


Figure 2-1. Current AASHTO S_r-N curves

Table 2-1
AASHTO Fatigue Categories

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
Plain Member	Base metal with rolled or cleaned surface. Flame-cut edges with ANSI smoothness of 1,000 or less.	T or Rev[a]	A	1,2
Built-Up Members	Base metal and weld metal in members of built-up plates or shapes (without attachments) connected by continuous full penetration groove welds (with backing bars removed) or by continuous fillet welds parallel to the direction of applied stress.	T or Rev	B	3,4,5,7
	Base metal and weld metal in members of built-up plates or shapes (without attachments) connected by continuous full penetration groove welds with backing bars not removed, or by continuous partial penetration groove welds parallel to the direction of applied stress.	T or Rev	B′	3,4,5,7
	Calculated flexural stress at the toe of transverse stiffener welds on girder webs or flanges.	T or Rev	C	6
	Base metal at ends of partial length welded coverplates with high-strength bolted slip-critical end connections. (See Note f.)	T or Rev	B	22
	Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends, or wider than flange with welds across the ends:			
	(a) Flange thickness ≤ 0.8 in.	T or Rev	E	7
	(b) Flange thickness > 0.8 in.	T or Rev	E′	7
	Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends.	T or Rev	E′	7
Groove Welded Connections	Base metal and weld metal in or adjacent to full penetration groove weld splices of rolled or welded sections having similar profiles when welds are ground flush with grinding in the direction of applied stress and weld soundness established by nondestructive inspection.	T or Rev	B	8,10
	Base metal and weld metal in or adjacent to full penetration groove weld splices with 2 ft radius transitions in width, when welds are ground flush with grinding in the direction of applied stress and weld soundness established by nondestructive inspection.	T or Rev	B	13
	Base metal and weld metal in or adjacent to full penetration groove weld splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½, with grinding in the direction of the applied stress, and weld soundness established by nondestructive inspection:			
	(a) AASHTO M 270 Grades 100/100W (ASTM A 709) base metal	T or Rev	B′	11,12
	(b) Other base metals	T or Rev	B	11,12
	Base metal and weld metal in or adjacent to full penetration groove weld splices, with or without transitions having slopes no greater than 1 to 2½, when the reinforcement is not removed and weld soundness is established by nondestructive inspection.	T or Rev	C	8,10,11,12
Groove Welded Attachments—Longitudinally Loaded[b]	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is less than 2 in.	T or Rev	C	6,15
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	15

(Sheet 1 of 4)

Note: Refer to AASHTO 1996 for Table 10.3.1A. For Figure 10.3.1C, see the last sheet of this table.

Table 2-1 (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is greater than 12 times the plate thickness or greater than 4 in.:			
	(a) Detail thickness < 1.0 in.	T or Rev	E	15
	(b) Detail thickness ≥ 1.0 in.	T or Rev	E′	15
	Base metal adjacent to details attached by full or partial penetration groove welds with a transition radius, R, regardless of the detail length:			
	—With the end welds ground smooth	T or Rev		16
	(a) Transition radius ≥ 24 in.		B	
	(b) 24 in. > Transition radius ≥ 6 in.		C	
	(c) 6 in. > Transition radius ≥ 2 in.		D	
	(d) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii without end welds ground smooth.	T or Rev	E	16
Groove welded Attachments—Transversely Loaded[b,c]	Detail base metal attached by full penetration groove welds with a transition radius, R, regardless of the detail length and with weld soundness transverse to the direction of stress established by nondestructive inspection:			
	—With equal plate thickness and reinforcement removed	T or Rev		16
	(a) Transition radius ≥ 24 in.		B	
	(b) 24 in. > Transition radius ≥ 6 in.		C	
	(c) 6 in. > Transition radius ≥ 2 in.		D	
	(d) 2 in. > Transition radius ≥ 0 in.		E	
	—With equal plate thickness and reinforcement not removed	T or Rev		16
	(a) Transition radius ≥ 6 in.		C	
	(b) 6 in. > Transition radius ≥ 2 in.		D	
	(c) 2 in. > Transition radius ≥ 0 in.		E	
	—With unequal plate thickness and reinforcement removed	T or Rev		16
	(a) Transition radius ≥ 2 in.		D	
	(b) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii with unequal plate thickness and reinforcement not removed.	T or Rev	E	16
Fillet Welded Connections	Base metal at details connected with transversely loaded welds, with the welds perpendicular to the direction of stress:			
	(a) Detail thickness ≤ 0.5 in.	T or Rev	C	14
	(b) Detail thickness > 0.5 in.	T or Rev	See Note[d]	
	Base metal at intermittent fillet welds.	T or Rev	E	—
	Shear stress on throat of fillet welds.	Shear	F	9
Fillet Welded Attachments—Longitudinally Loaded[b,c,e]	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress, is less than 2 in. and stud-type shear connectors.	T or Rev	C	15,17,18,20
	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress, between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	15,17
	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress greater than 12 times the plate thickness or greater than 4 in.:			
	(a) Detail thickness < 1.0 in.	T or Rev	E	7,9,15,17
	(b) Detail thickness ≥ 1.0 in.	T or Rev	E′	7,9,15

(Sheet 2 of 4)

Table 2-1 (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	Base metal adjacent to details attached by fillet welds with a transition radius, R, regardless of the detail length:			
	—With the end welds ground smooth (a) Transition radius ≥ 2 in. (b) 2 in. > Transition radius ≥ 0 in.	T or Rev	 D E	16
	—For all transition radii without the end welds ground smooth.	T or Rev	E	16
Fillet Welded Attachments—Transversely Loaded with the Weld in the Direction of Principal Stress[b,c]	Detail base metal attached by fillet welds with a transition radius, R, regardless of the detail length (shear stress on the throat of fillet welds governed by Category F):			
	—With the end welds ground smooth (a) Transition radius ≥ 2 in. (b) 2 in. > Transition radius ≥ 0 in.	T or Rev	 D E	16
	—For all transition radii without the end welds ground smooth.	T or Rev	E	16
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip resistant connections, except axially loaded joints which induce out-of-plane bending in connecting materials.	T or Rev	B	21
	Base metal at net section of high-strength bolted bearing-type connections.	T or Rev	B	21
	Base metal at net section of riveted connections.	T or Rev	D	21
Eyebar or Pin Plates	Base metal at the net section of eyebar head, or pin plate	T	E	23, 24
	Base metal in the shank of eyebars, or through the gross section of pin plates with: (a) rolled or smoothly ground surfaces (b) flame-cut edges	 T T	 A B	 23, 24 23, 24

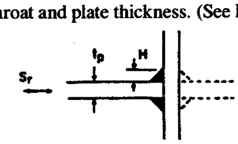
[a] "T" signifies range in tensile stress only, "Rev" signifies a range of stress involving both tension and compression during a stress cycle.

[b] "Longitudinally Loaded" signifies direction of applied stress is parallel to the longitudinal axis of the weld. "Transversely Loaded" signifies direction of applied stress is perpendicular to the longitudinal axis of the weld.

[c] Transversely loaded partial penetration groove welds are prohibited.

[d] Allowable fatigue stress range on throat of fillet welds transversely loaded is a function of the effective throat and plate thickness. (See Frank and Fisher, Journal of the Structural Division, ASCE, Vol. 105, No. ST9, Sept. 1979.)

$$S_r = S_r^C \left(\frac{0.06 + 0.79H/t_p}{1.1 t_p^{1/6}} \right)$$



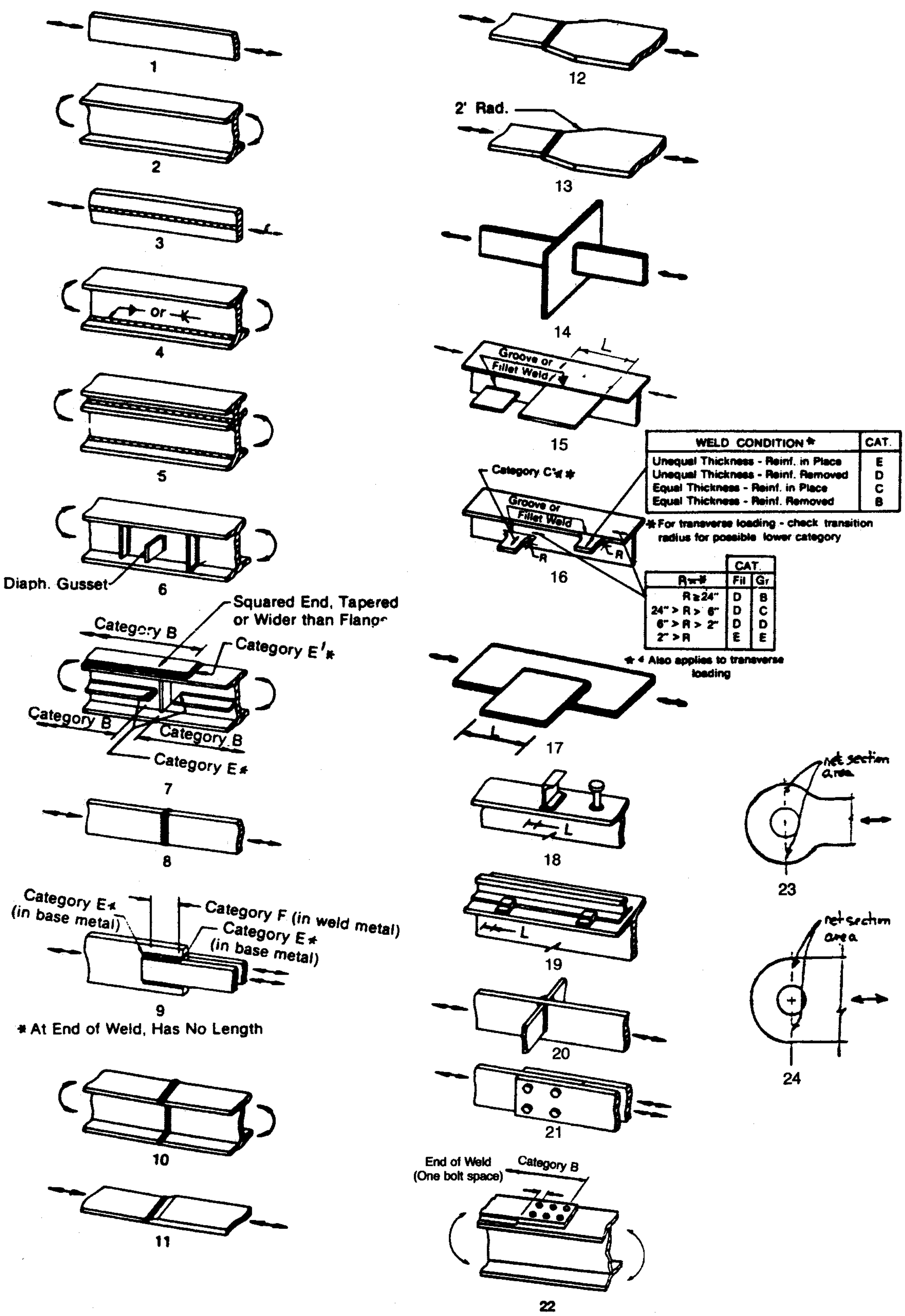
where S_r^C is equal to the allowable stress range for Category C given in Table 10.3.1A. This assumes no penetration at the weld root.

[e] Gusset plates attached to girder flange surfaces with only transverse fillet welds are prohibited.

[f] See Wattar, Albrecht and Sahli, Journal of Structural Engineering, ASCE, Vol. III, No. 6, June 1985, pp. 1235–1249.

(Sheet 3 of 4)

Table 2-1 (Concluded)



(Sheet 4 of 4)

(2) The American Institute of Steel Construction (AISC) has adopted AASHTO S_r-N curves for fatigue design (AISC 1989, 1994). The AWS has also adopted the S_r-N approach for design of welded structures and has published S_r-N curves and guidelines for categorization of welded details for redundant and nonredundant structural members (ANSI/AWS D1.1). The AWS S_r-N requirements vary slightly from those of AASHTO, which are adopted herein.

c. Fatigue strength of riveted structures.

(1) Fisher et al. (1987) compiled all the published data from fatigue testing of full-size riveted members. Based on these data, the fatigue strength of riveted members is relatively insensitive to the rivet pattern or type of detail (cover plate details, longitudinal splice plates, and angles or shear-splice details). The data are plotted in Figure 2-2 with the AASHTO fatigue strength (S_r-N) curves of Categories C and D, which have been developed for welded details. Based on the data shown in Figure 2-2, it is recommended that Category D be assumed for structural details in riveted members subjected to stress ranges higher than 68.95 MPa ($S_r \geq 68.95$ MPa (10 ksi)), and Category C be assumed for the lower stress range, high-cycle region. This recommendation is similar to the current American Railway Engineers Association (AREA) standards (AREA 1992). In cases where there are missing rivets or a significant number of rivets have lost their clamping force, Category E or E' strength should be assumed.

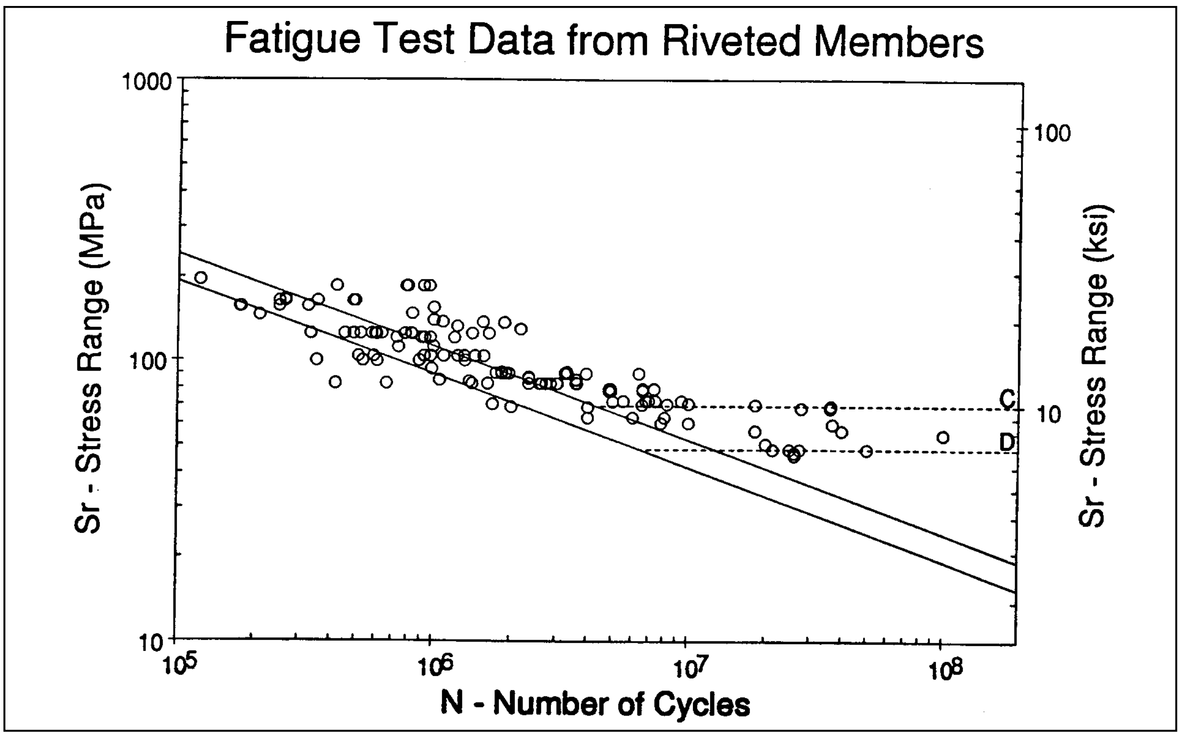


Figure 2-2. Fatigue test data from full-size riveted members

(2) There are insufficient data for a conclusion about the fatigue limit of riveted members. Fisher et al. (1987) state that no fatigue failure has ever occurred when the stress range was below 41.3 MPa (6 ksi) provided that the member or detail was not otherwise damaged or severely corroded.

(3) A major advantage of riveted (or bolted) members is that they are internally redundant. Cracking that propagates from a rivet hole is the typical phenomenon of fatigue damage of riveted members as shown in Figures 2-3 and 2-4. Since cracks usually do not propagate from one component into adjacent components, fatigue cracking in riveted members is not continuous as in welded members. In other words, fatigue cracking in one component of a riveted structural member usually does not cause the complete failure of the member.

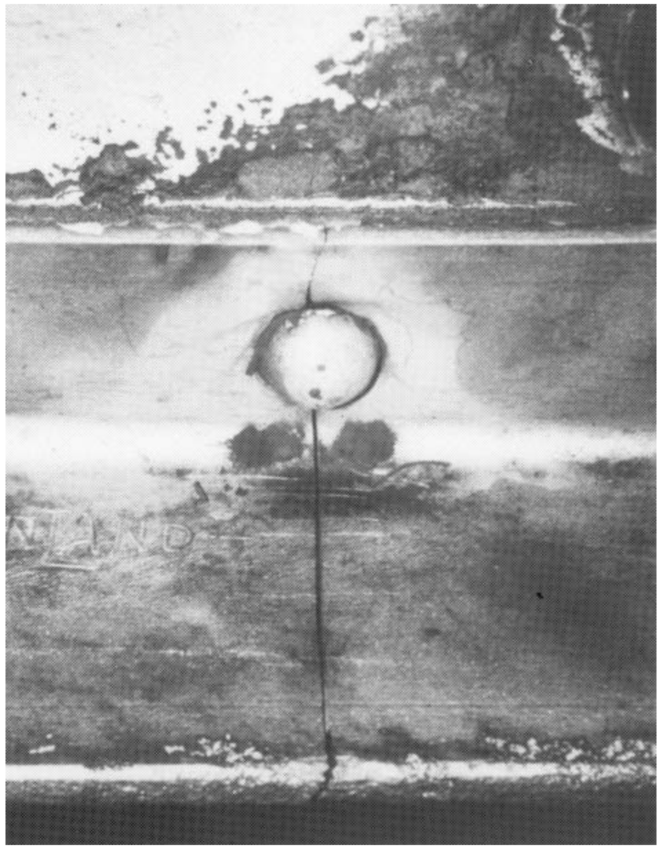


Figure 2-3. Typical fatigue cracking of riveted member

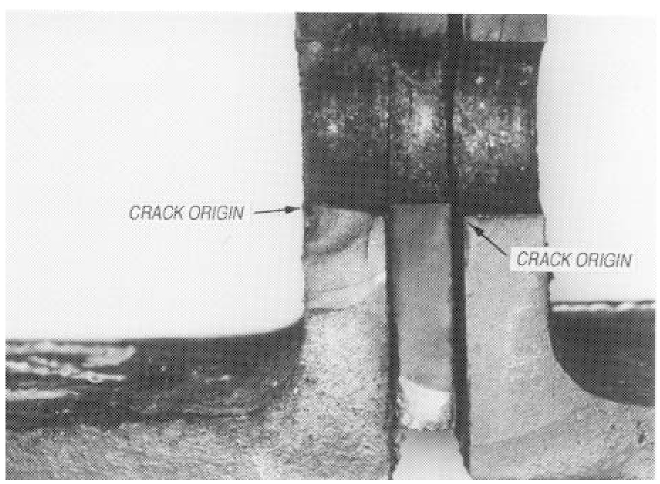


Figure 2-4. Crack surface at the edge of rivet hole

Therefore, fatigue cracks would more likely be detected long before the load-carrying capacity of the riveted member is exhausted.

d. Fatigue strength of corroded members. For severely corroded members where corrosion notching has occurred, Category E or E' curves and the corresponding fatigue limits have been suggested for cases. When corrosion is severe and notching occurs, a fatigue crack may initiate from the corroded region as shown in Figure 2-5. In cases where corrosion has resulted in loss of more than 20 percent of the cross section, the corresponding increase in stress should also be considered.

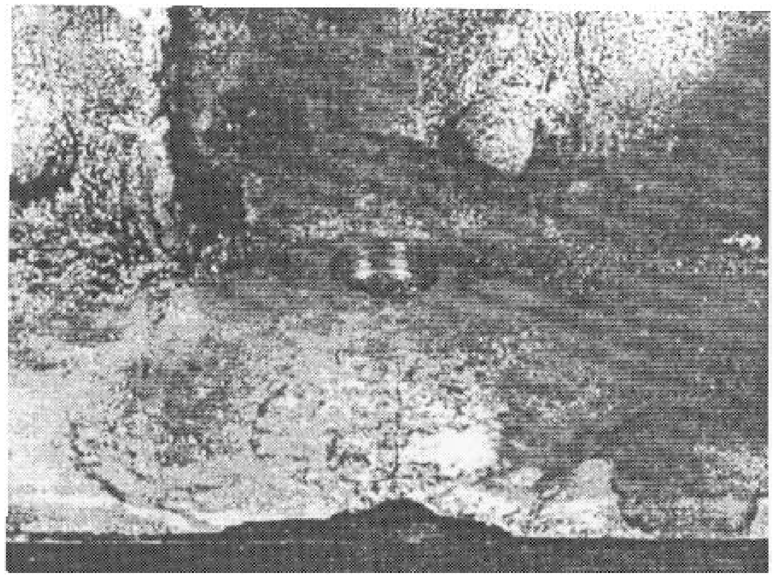


Figure 2-5. Fatigue crack from corrosion notch into rivet hole

e. Variable-amplitude fatigue loading.

(1) Most of the fatigue test data and the S_r-N curves in Figures 2-1 and 2-2 were established from constant-amplitude cyclic loads. In reality, however, structural members are subjected to variable-amplitude cyclic loads resulting in a spectrum of various stress ranges. Variable-amplitude fatigue loading may occur on hydraulic steel structures.

(2) In order to use the available S_r-N curves for variable-amplitude stress ranges, an equivalent constant-amplitude stress range S_{re} can be determined from a histogram of the stress ranges (Figure 2-6). S_{re} is calculated as the root-mean-cube of the discrete stress ranges S_{ri}

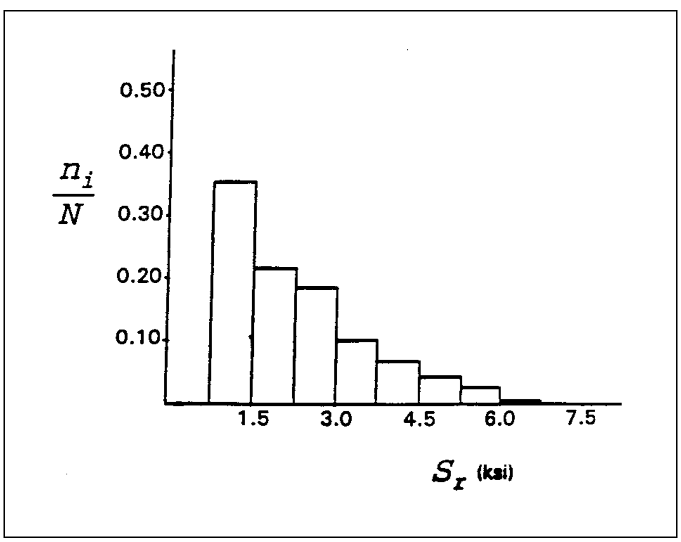


Figure 2-6. Sample stress range histogram

$$S_{re} = \sqrt[3]{\sum_{i=1}^{m} \frac{n_i S_{ri}^3}{N}} \tag{2-3}$$

where

m = number of stress range blocks

n_i = number of cycles corresponding to S_{ri}

S_{ri} = magnitude of a stress range block

f. Repeated loading for hydraulic steel structures. The general function of hydraulic steel structures is to dam and control the release of water. Sources of repeated loading include changes in load due to pool fluctuations, operation of the hydraulic steel structure, flow-induced vibration, and wind and wave action.

(1) Operation.

(a) Spillway gates. During the routine operation of actuating a spillway gate, cyclic loads are applied to structural members due to the change in hydrostatic pressure on the structure as the gate is raised and then lowered. Although this load case has the potential to produce large variation of stress in structural components, the frequency of occurrence (a very conservative assumption is one cycle per day) is too low to cause fatigue damage. One lifting/lowering operation per day results in only 18,000 cycles in a 50-year life. This is well below the number of cycles necessary for consideration of fatigue. Consequently, the possibility that repeated loads in spillway gates due to operations would cause fatigue damage is unlikely.

(b) Lock gates. Repeated loading for various structural components occurs due to variation in the lock chamber water level and to opening and closing of gates. The number of load cycles is a function of the number of lockages that occurs at the lock. The number of load cycles due to gate operation or filling/emptying the lock chamber per lockage varies between 0.5 and 1.0 depending on barge traffic patterns. Gates at busy locks can easily endure greater than 100,000 load cycles within a 50-year life. Therefore, fatigue loading is significant and must be considered in design and evaluation.

(2) Flow-induced vibration. This phenomenon may produce significant cyclic loads on hydraulic steel structures because of the potential for the occurrence of high-frequency live load stresses above the fatigue limit. Spillway gates especially can experience some level of flow-induced vibration whenever water is being discharged, but severe vibration usually occurs only when the gate is open at a certain position. Vibration of tainter gates is heavily influenced by flow conditions (i.e., gate opening and tailwater elevation) and bottom seal details. Approximate measurements have indicated that a frequency of vibration of 5-10 Hz is reasonable (Bower et al. 1992). This frequency is large enough to cause fatigue damage in a short time even for relatively low stress range values. Although a hydraulic steel structure would rarely be operated in such a position for any length of time, flow-induced vibration should be considered as a possible source of fatigue loading. An example of the fatigue evaluation of a spillway gate including vibration loading is given in Chapter 7.

(3) Wind and wave action. This is a continuous phenomenon that has not caused fatigue problems in hydraulic steel structures probably due to the low magnitude of stress range for normal conditions.

2-4. Design Deficiencies

Many existing hydraulic steel structures were designed during the early and mid-1900's. Analysis and design technologies have significantly improved, producing the current design methodology. Original design loading conditions may no longer be valid for the operation of the existing structure, and overstress conditions may exist. Current information, including modern welding practice and fatigue and fracture control in structures, was not available when many of the initial designs were performed. Consequently, low category fatigue details and low toughness materials exist on some hydraulic steel structures. In addition, the amount of corrosion anticipated in the original design may not accurately reflect actual conditions, and structural members may now be undersized. To evaluate existing structures properly, it is important that the analysis and design information for the structure be reviewed to assure no design deficiencies exist.

2-5. Fabrication Discontinuities

a. For strength and economic reasons, EM 1110-2-2703 recommends that hydraulic steel structures be fabricated using structural-grade carbon steel. Standards such as ASTM A6/A6M or ASTM A898/A898M have been developed to establish allowable size and number of discontinuities for base metal used to fabricate hydraulic steel structures. In addition, EM 1110-2-2703 also recommends that the steel structures be welded in accordance with the Structural Welding Code-Steel (ANSI/AWS D1.1). This code provides a standard for limiting the size and number of various types of discontinuities that develop during welding. Although these criteria exist, when a hydraulic steel structure goes into service, it does contain discontinuities.

b. Discontinuities that exist during initial fabrication are rejectable only when they exceed specified requirements in terms of type, size, distribution, or location as specified by ANSI/AWS D1.1. Welded fabrication can contain various types of discontinuities that may be detrimental (see paragraph 2-2). This is especially important when considering weldments involving thick plates, because thick plates are inherently less tough and welding residual stresses are high.

c. Frequently, plates 38 mm (1-1/2 in.) in thickness and greater are used as primary welded structural components on hydraulic steel structures. It is not uncommon to see such thick plates used as flanges, embedded anchorage used to support hydraulic steel structures, hinge and operating equipment connections, diagonal bracing, lifting or jacking assemblies, or platforms to support operating equipment that actuates the hydraulic steel structures. In addition, thick castings such as sector gears used for operating such structures as lock gates may be susceptible to brittle fracture. Hydraulic steel structures have experienced cracking during fabrication and after the thick assemblies are welded and placed into service.

2-6. Operation and Maintenance

Proper operation and maintenance of hydraulic steel structures are necessary to prevent structural deterioration. The following items are possible causes of structural deterioration that should be considered:

a. Weld repairs are often sources of future cracking or fracture problems, particularly if the existing steel had poor weldability as is often the case with older gates.

b. If moving connections are not lubricated properly, the bushings will wear and result in misalignment of the gate. The misalignment will subsequently wear contact blocks and seals, and unforeseen loads may develop.

c. Malfunctioning limit switches could result in detrimental loads and wear.

d. A coating system or cathodic protection that is not maintained can result in detrimental corrosion.

e. Loss of prestress in the gate leaf diagonals reduces the torsional stability of miter gates during opening and closing.

f. Proper maintenance of timber fenders and bumpers is necessary to provide protection to the gate and minimize deterioration.

2-7. Unforeseen Loading

a. Accidental overload or dynamic loading of a gate can result in deformed members or fracture. When structural members become plastically deformed or buckled, they may have significantly reduced strength and/or otherwise impair the performance of a hydraulic steel structure. The extent and nature of any noticeable plastic deformation should be noted and accurately described during the inspection process, and its effect on the performance of the structure should be assessed in the ensuing evaluation as further discussed in Chapter 6. Fractures that occur must generally be repaired. Considerations for repair are discussed in Chapter 8.

b. Dynamic loading due to hydraulic flow and impact loading due to vessel collision are currently unpredictable. The dynamic loading may be caused by hydraulic flow at the seals or may occur when lock gates are used to supplement chamber filling or skim ice and debris. Impact loading can occur from malfunctioning equipment on moving vessels or operator error. Fracture likelihood is enhanced with dynamic loads, since the fracture toughness for steels decreases with increasing load rate. Other unusual loadings may occur from malfunctioning limit switches or debris trapped at interfaces between moving parts. It is also possible that unusual loads may develop on hydraulic steel structures supported by walls that are settling or moving. These unusual loads can cause overstressing and lead to deterioration.